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**APPLICATION OF A MEMORY SURFACE MODEL TO PREDICT WHOLE-LIFE  
SETTLEMENTS OF A SLIDING FOUNDATION**

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### **ABSTRACT**

In this paper a novel modelling procedure is proposed to estimate whole-life settlements of tolerably mobile sliding foundations. A new kinematic hardening-critical state-state parameter constitutive model, the Memory Surface Hardening model, is implemented in a one-dimensional analysis to predict accumulated vertical settlements under drained lateral cyclic loading. The Memory Surface Hardening model performance is compared with the Modified Cam Clay and Severn-Trent Sand models. The Memory Surface Hardening model is adopted to simulate available experimental data from centrifuge tests to predict the settlement of a sliding foundation at the final stable state (i.e. no further volume changes occur).

**Keywords: Settlement; cyclic loading; offshore engineering; soil modelling; memory surface.**

# 1    **1    Introduction**

2    Sliding foundations are a novel concept to meet the increasingly challenging demand to limit  
3    the footprint of subsea mudmats. In contrast to the traditional paradigm that foundations  
4    remain stationary and resist all the applied loads, sliding foundations are designed to move  
5    tolerably across the seabed to relieve some of the applied loads, thus requiring a smaller  
6    footprint. Sliding displacements are caused, and also limited, by expansion and contraction of  
7    attached pipelines ([1], [2], [3]). In general, magnitudes of displacement are sufficient to  
8    cause shear failure between the foundation and the soil, where the mobilised ratio of shear  
9    stress to normal effective stress is greatest.

10   Subsea mudmats are shallow, mat-style foundations used to support pipeline infrastructure  
11   for offshore hydrocarbon developments. Foundation loads derive from the self-weight of the  
12   mat, the supported structure and thermal expansion and contraction of the attached pipelines.  
13   Increasing operational loads coupled with softer seabeds has resulted in traditional subsea  
14   mudmat designs exceeding the installation capacity of pipelaying vessels. The expense of an  
15   additional heavy lift vessel on site to install over-sized mudmats can be prohibitive. Sliding  
16   foundations offer a potential solution to this impasse ([1], [2], [3]).

17   Observations of performance of a sliding foundation on soft clay from a programme of  
18   centrifuge model tests are reported by Cocjin et al. [2]. The considered sliding mudmat  
19   comprised a rectangular rough-based mat of breadth to length aspect ratio of 0.5 and was  
20   provided with edge ‘skis’ to facilitate sliding (rather than overturning that may lead to  
21   overstressing of the pipeline connections). A schematic representation of the generalized  
22   geometry is presented in Fig. 1, also showing an attached pipeline connection. The tests  
23   involved a number of cycles of undrained sliding with intervening periods of consolidation.  
24   The model data showed settlement of the mat during each period of consolidation resulting  
25   from dissipation of shear induced pore pressures generated during the preceding sliding

26 event. The accumulated mat settlements reduced with each slide, ultimately reaching a stable  
27 state condition with no further volume change in the soil. This stable state was shown to be  
28 equivalent to the drained state [4].

29

30 This last observation is illustrated in Fig. 2, through an analysis of a strain-imposed cyclic  
31 simple shear test under constant total vertical stress conditions using the Modified Cam Clay  
32 model [5]. Results compare the stress-volume changes under drained cycles of loading and  
33 undrained cycles of loading with intervening periods of consolidation. It is evident that the  
34 volumetric behaviours are comparable and the final stable state from the two simulations  
35 converge. A further check is performed by comparing the variation of the void ratio with the  
36 number of cycles for the performed simulations. The trends are similar, which confirms that  
37 the soil response is comparable.

38 It can therefore be surmised that consideration of drained sliding and associated (drained)  
39 volumetric strain is an appropriate approximation for undrained generation and subsequent  
40 dissipation of shear induced excess pore pressures. On this assumption, this study investigates  
41 the volumetric response of drained lateral cyclic loading of a sliding foundation. Three  
42 constitutive models are adopted to estimate vertical settlements over the whole-life of a  
43 sliding foundation; predicted results are compared with available data from centrifuge tests  
44 performed at the University of Western Australia – Centre for Offshore Foundation Systems  
45 (UWA-COFS) [2].

46

## 47 **2 Analysis set up and soil model**

48

## 49 **2.1 Analysis set up**

50 The framework of the 1-D analyses considering a sliding mudmat of width  $B$  resting on half-  
51 space soil is shown in Fig. 3. The overall soil response under the shearing imposed by the  
52 sliding mudmat is computed through a layer-by-layer summation, by dividing the half-space  
53 soil domain into  $n$  different layers of thickness  $h_i$ , where  $i$  is the layering index, up to the  
54 depth of influence of the loading imposed by the sliding mudmat. The average response of  
55 the individual layer  $i$  is computed at a characteristic soil element located at its vertical mid-  
56 point, as shown in Fig. 3. Since the large stress gradient in the layers close to the foundation  
57 base may affect the accuracy of layer-average based procedure, it is customary to use thinner  
58 layering in the uppermost layers. The loading conditions in each layer was approximated  
59 using available elastic solutions for vertical and shear load distribution with depth [6], while  
60 the soil response was computed assuming the three specific elasto-plastic constitutive models  
61 outlined above (and described in detail below). Comparison of the shear stress distribution  
62 beneath a surface foundation under horizontal sliding in an elasto-plastic medium determined  
63 from a finite element analysis (described in [7]) showed adoption of an elastic shear stress  
64 distribution to be an appropriate simplification (Figure 4). Each elasto-plastic model was  
65 coded in Matlab and solved incrementally using the fourth order Runge-Kutta numerical  
66 integration method.

67

## 68 **2.2 Constitutive soil models**

69 Three soil constitutive models were used to predict the accumulated settlement during  
70 drained lateral cyclic loading: the Modified Cam Clay model (MCC) [5], the Severn-Trent  
71 Sand model (STS) [8-9] and the Memory Surface Hardening model (MSH) [10-12], which  
72 are represented schematically in Fig. 5. All the three elasto-plastic models assume isotropic  
73 elastic laws and are constructed within the framework of Critical State Soil Mechanics

74 [13,14]. The critical state is modelled as a straight line in the void ratio-mean effective  
75 pressure semi-logarithmic plane ( $e-\ln p'$ ). The deviatoric section of all the yield surfaces  
76 follows the shape proposed by Argyris et al. (1974) [15] which avoids the presence of  
77 singularities and ensures that the critical state strength varies with the Lode angle. The main  
78 differences between the models are: a) the shape of the model surfaces in the  $q-p'$  plane,  
79 which are ellipses for the MCC and wedges for the other two models (STS and MSH), b) the  
80 adopted dilatancy rule (defined by  $d$  in Fig. 5) and, most importantly, c) the number of model  
81 surfaces (defined by their slope  $\eta$  in Fig. 5) which increases from the simplest MCC model  
82 using only a yield surface, to the most sophisticated MSH model, which postulates the  
83 existence of yield, bounding and memory surfaces. The larger number of model surfaces  
84 implies an increased complexity of the employed hardening rule which can allow a more  
85 accurate simulation of the non-linear stress and density dependent behaviour of soil as well as  
86 the influence of past stress history. This last aspect is expected to be the most important to  
87 accurately predict the cyclic response of the sliding mudmat in the following analyses. A  
88 brief description of each of the three soil models is provided in the following alongside  
89 schematic representations in Fig. 5. The most fundamental mathematical relationships are  
90 also reported in Table 1. The full mathematical formulation for each model can be found in  
91 the relevant referenced literature as referenced below.

92

93 *Modified Cam Clay (MCC) model* [5]: An elasto-plastic model that assumes an elliptical  
94 yield locus that passes through the origin of the stress plane and bounds soil elastic states  
95 (Fig. 5a). The size of the yield locus is governed by the parameter  $p_c$ . Under elasto-plastic  
96 loading conditions, the yield locus evolves preserving its shape and its intersection with the  
97 origin of the stress state axis, with the consequent variation in size of the elastic region. An  
98 associative flow rule is assumed.

100 *Severn-Trent Sand (STS) model* [8,9]: A kinematic hardening – bounding surface elasto-  
101 plastic constitutive model. A purely elastic region is bounded by the yield surface ( $f_Y$ ) which  
102 is an open wedge with its apex at the origin of the  $q$ - $p'$  stress axes (Fig. 5b) and moves in the  
103 stress space (kinematic hardening) under shearing. The bounding surface ( $f_B$ ) represents the  
104 current soil strength and its size is influenced by the current soil density through the state  
105 parameter [16]. Based on the bounding surface theory [17,18], plastic soil stiffness is  
106 assumed to depend on the distance between the current stress state  $\sigma$  and the conjugate one on  
107 the bounding surface  $\sigma^B$ , as shown in Fig. 5 (b) and in Table 1. Upon unloading-reloading,  
108 the soil response is initially fully elastic inside the yield surface but elasto-plastic conditions  
109 are invoked when the opposite boundary of the yield locus is reached.

110

111 *Memory Surface Hardening (MSH) model* [10-12]: A recently developed constitutive model  
112 based on an extension of the STS model by introducing an additional surface, the memory  
113 surface ( $f_M$ ), which retains information of the recent stress history (Fig. 5c). The memory  
114 surface bounds a region of stress states that the soil has already experienced. When the  
115 current stress state lies inside the memory surface, the memory surface  $f_M$  acts as an  
116 additional bounding surface so that the plastic soil modulus is governed by an additional  
117 hardening term depending on the distance between the current stress ( $\sigma$ ) state and its  
118 projection on the memory surface ( $\sigma^M$ ). This results in a stiffer soil behaviour during repeated  
119 loading compared with virgin loading conditions. The memory surface can evolve in size and  
120 translate in the stress space according to three rules:

121

122 Rule 1: Changes in size of the memory surface are linked to plastic strains;



123 Rule 2: The current stress state must always lie inside or on the boundary of the memory  
 124 surface;

125 Rule 3: The yield locus must always be enclosed within the memory surface.

126 **Table 1: Summary of functions used in the selected constitutive models**

	Modified Cam Clay [5]	Severn-Trent Sand [8,9]	Memory Surface Hardening [10, 12]
<b>Yield surface</b>	$f_Y = \frac{q^2}{M^2} + p'(p' - p_c)$	$f_Y = q - \eta^Y p'$	$f_Y = q - \eta^Y p'$
<b>Bounding surface</b>	---	$f_B = q - \eta^B p'$	$f_B = q - \eta^B p'$
<b>Memory surface</b>	---	---	$f_M = q - \eta^M p'$
<b>Flow rule</b>	$d = \frac{M^2 - \eta^2}{2\eta}$	$d = A_d[(1 + k_d)M - \eta]$	$d = A_d[(1 + k_d)M - \eta]$
<b>Hardening modulus</b>	$H = p' \left( \frac{v p'_c}{\lambda - \kappa} \right) (2p' - p'_c)$	$H = \frac{b^2}{b_{max}\beta}$	$H = \frac{b^2}{b_{max}\beta} \exp \left[ \frac{\mu(1-k\psi)b^M}{b} \left( \frac{p'}{p_{ref}} \right)^{0.5} \right]$

127

128

129

130 The hardening modulus  $H$  presented in Table 1 is introduced to calculate the elasto-plastic  
 131 stiffness matrix. Following [8,9], this is calculated as

132

$$133 \quad \mathbf{D}^{ep} = \mathbf{D}^e - \frac{\mathbf{D}^e \mathbf{m} \mathbf{n}^T \mathbf{D}^e}{\mathbf{n}^T \mathbf{D}^e \mathbf{m} + H} \quad (1)$$

134

135 Where  $\mathbf{D}^e$  is the elastic stiffness matrix,  $\mathbf{n}$  is the normal outwards from the yield surface and  
 136  $\mathbf{m}$  is the normal outwards from the plastic potential surface. An exhaustive description of the  
 137 terms is provided in [8,9] and [12]. It should be noted the slightly different formulation of the  
 138 hardening modulus of the Memory Surface Hardening model from the one defined in [10-12],  
 139 where the dependence on the ratio  $(p'/p_{ref})^{0.5}$  is inspired by the previous work from Hardin  
 140 and Black [19] and it accounts for the influence of the effective mean stress. The quantity  $b$

141 represents the distance between the current stress state ( $\sigma$ ) and the image stress on the  
142 bounding surface ( $\sigma^B$ ) with  $b_{max}$  its maximum value, the quantity  $b^M$  is the distance between  
143 the current stress state ( $\sigma$ ) and the image on the memory surface ( $\sigma^M$ ),  $\psi$  is the state  
144 parameter,  $p'$  is the effective mean pressure,  $p_{ref}$  is a reference pressure ( $p_{ref} = 100$  kPa) and  $\mu$   
145 and  $\beta$  are two constitutive parameters. The constitutive parameters  $\beta$  and  $\mu$  are quite relevant  
146 for this application because they govern the magnitude of the accumulated strains during the  
147 first slide and under cyclic loading conditions, respectively. It should be noted that by  
148 imposing the parameter  $\mu=0$ , the hardening modulus of the STS is re-established.

149

### 150 **3 Calibration of the constitutive parameters**

151 Summary and description of the constitutive parameters for the three models considered are  
152 provided in Table 2. The number of constitutive parameters required by the models is  
153 proportional to their complexity - 5, 10 and 12 for the MCC, STS and MSH respectively –  
154 but, as shown in Table 2, the fundamental parameters are shared and the increased  
155 complexity is reflected only in the use of additional parameters. For consistency in the  
156 following analysis, the shared constitutive parameters must also assume the same value for  
157 the different models employed. The calibrated numerical values of the model parameters are  
158 also provided in Table 2. The first five constitutive parameters in Table 2 are shared by all  
159 three models and have been calibrated in accordance with Stewart [20] and Acosta-Martinez  
160 and Gourvenec [21], who used the MCC model to simulate the behaviour of exactly the same  
161 kaolin clay material used in the centrifuge model tests [2] back-analysed in this paper. Thus,  
162 only the calibration procedure for the other constitutive parameters (no. 6 to 12 in Table 2)  
163 relative to the STS and MSH models is presented in this section. The constitutive parameters  
164 of the STS and MSH models have been calibrated against an available drained cyclic triaxial  
165 test on speswhite kaolin clay [22]. Information on the mechanical properties of this material

166 can be found in [23] and shows the kaolin is similar to that used in the centrifuge tests [2].  
 167 The shared constitutive parameters between the STS and MSH (No. 6 to 10 in Table 2) have  
 168 been calibrated by fitting the initial virgin behaviour while the constitutive parameter  $\mu$  has  
 169 been calibrated by fitting the experimental data under the subsequent cyclic conditions. The  
 170 damage parameter  $\zeta$  of the MSH model governing an eventual damage of the memory surface  
 171 ( $f_M$ ) upon dilation is not included because it is not relevant for the very soft soil conditions  
 172 simulated in this study. The model calibration against an available cyclic drained triaxial test  
 173 is provided in Fig. 6.

174 **Table 2: Constitutive parameters description.**

Constitutive model			No.	Parameter	
Memory Surface Hardening model	Severn-Trent Sand model	Modified Cam Clay model	1	$\kappa$	0.044
			2	$M_{cv}$	0.92 ( $\phi'=23.5^\circ$ )
			3	$e_{CSL}$	2.14
			4	$\lambda$	0.205
			5	$\nu$	0.3
			6	$R$	0.05
			7	$\beta$	0.017
			8	$k^*$	1.5
			9	$A_d$	2.5
			10	$k_d$	0
			11	$\mu$	11.5
			12	$\zeta$	n. r.

175

## 176 **4 Analysis procedure**

177 In accordance with the centrifuge tests [2], the analyses are referred to a rectangular mudmat  
178 of width  $B = 5$  m and aspect ratio  $B/L = 0.5$ , where  $L$  is the foundation length. The soil  
179 beneath the sliding foundation has been discretised in  $n = 13$  soil layers with soil layers  
180 thickness being  $h_i = 0.5$  m for the two shallowest layers and  $h_i = 1$  m for the deeper ones. The  
181 full loading history of the soil has been considered, including the following three main  
182 loading stages:

183

- 184 1) Geostatic stage (soil self-weight consolidation prior to installation of foundation).
- 185 2) Foundation set-down stage (consolidation after foundation installation).
- 186 3) Foundation sliding stage (cyclic sliding of foundation).

187

### 188 **4.1 Geostatic stage**

189 The in situ soil state at the end of soil self-weight consolidation prior to installation of the  
190 foundation can be represented as  $K_0$ -consolidation process under axisymmetric stress  
191 conditions. This will be referred to as condition '0' as it represents the initial soil state before  
192 installation of the mudmat. Stress variables referring to this condition are denoted by the  
193 subscript '0'. The initial in situ vertical effective stress is given by:

194

$$195 \sigma'_{v0} = \gamma' z \quad (2)$$

196

197 where  $\gamma'$  is the drained unit weight of the kaolin, taken as  $\gamma' = 5.7$  kN/m<sup>3</sup> [2]. The initial in  
198 situ horizontal effective stress is given by:

199

$$200 \sigma'_{h0} = K_0 \sigma'_{v0} \quad (3)$$

201

202 where  $K_0$  is the lateral earth pressure coefficient given by Jaky's formula [24]:

203

$$204 \quad K_0 = 1 - \sin\phi' \quad (4)$$

205

206 with the critical state friction angle  $\phi'$  as assumed in Table 2.

207 The initial void ratio is estimated using the MCC model formulation and imposing normally  
208 consolidated conditions:

209

$$210 \quad e_0 = e_{CSL} - (\lambda - \kappa) \ln \frac{p'_{c0}}{2} - \kappa \ln p'_0 \quad (5)$$

211

212 where  $p'_0$  is the current in situ effective mean pressure determined using  $\sigma'_{v0}$  and  $\sigma'_{h0}$ . The  
213 quantity  $p'_{c0}$  is the pre-consolidation pressure defining the size of the MCC yield locus that  
214 can

215 be derived using the formulation of the yield locus for the MCC model:

216

$$217 \quad p'_{c0} = p'_0 + \frac{q_0^2}{p'_0 M^2} \quad (6)$$

218

219 where  $q_0 = \sigma'_{v0} - \sigma'_{h0}$  is the deviator stress. The estimated initial in situ vertical and horizontal  
220 effective stress profiles,  $\sigma'_{v0}$  and  $\sigma'_{h0}$ , and void ratio profile  $e$  with depth  $z$  are shown in Fig.  
221 7a-b respectively.

222

## 223 **4.2 Foundation installation stage**

224 The change in vertical effective stress  $\Delta\sigma'_v$  with depth  $z$  resulting from placement of the  
225 foundation load is approximated with the elastic solution of Poulos & Davis (1974) [6]. The

226 stress increase below the centre of a rectangular foundation induced by a uniform vertical  
 227 pressure  $q_{op}$  can be expressed as:

228

$$229 \quad \Delta\sigma'_v = 2 \frac{q_{op}}{\pi} \left[ \frac{B}{2A} \left( 1 + \frac{z^2}{2A^2} \right) \operatorname{atan} \frac{L}{2A} + \frac{L}{2D} \left( 1 + \frac{z^2}{2D^2} \right) \operatorname{atan} \frac{B}{2D} + \frac{BLz^2}{8C^2} \left( \frac{1}{A^2} + \frac{1}{D^2} \right) \right] \quad (7)$$

230

231 where  $A, B, C, D, L$  and  $z$  are geometrical distances represented schematically in Fig. 8.

232 A schematic representation of the normalized vertical stress change  $\Delta\sigma'_v/q_{op}$  with depth  $z$  is  
 233 represented in Fig. 9a. At this stage, the value of the lateral earth pressure coefficient  
 234 assumed in Eq. (4) was compared to the stress ratio calculated assuming a horizontal stress  
 235 change from the elastic solution of Poulos & Davis (1974) [6]. At a normalized depth  $z/B$  of  
 236 0.1, a difference of 15% was found, with the  $K_0$  value calculated in Eq. (4) being larger than  
 237 the horizontal stress ratio calculated using the elastic solution. This difference reduced to  
 238 10% at a normalised depth  $z/B = 0.3$ .

239 The magnitude of the foundation vertical pressure  $q_{op}$  is a variable (input) parameter and can  
 240 be defined as a portion of the available undrained bearing capacity  $q_u$ . In field conditions for  
 241 subsea mudmats on soft clays, the ratio  $q_{op}/q_u$  is generally between 0.3 and 0.5 [2]. For  
 242 consistency with the centrifuge tests [2], the foundation vertical pressure  $q_{op}$  is set to satisfy  
 243 the following condition:

244

$$245 \quad q_{op} = 0.3q_u \quad (8)$$

246

247 To ensure consistency between the calculated bearing capacity and the constitutive models,  
 248 the undrained shear strength profile is determined from the assumed elastic and critical state  
 249 soil properties. Following the procedure outlined in [25], it is possible to determine the

250 undrained shear strength profile using the input parameters of the MCC model as shown in  
 251 Eqs (9) to (11).

$$252 \quad \frac{s_u}{\sigma'_{v0}} = g(\theta) \cos \theta \frac{1+2K_0}{3} \left[ \frac{(a^2+1)}{2} \right]^{1-\left(\frac{\kappa}{\lambda}\right)} \quad (9)$$

253 where

$$254 \quad g(\theta) = \frac{\sin \phi'}{\cos \theta + \left(\frac{1}{\sqrt{3}}\right) \sin \phi' \sin \theta} \quad (10)$$

255 and

$$256 \quad a = \frac{\sqrt{3}(1-K_0)}{g(-30^\circ)(1+2K_0)} \quad (11)$$

257 The symbol  $\theta$  is the Lode's angle, taken as 0 to represent plane strain shear strength. The  
 258 undrained shear strength profile is calculated by assuming the same undrained shear strength  
 259 at the surface  $s_{um} = 0.52$  kPa as that measured in situ; to do this, an extra overburden stress of  
 260 1.85 kPa is applied in situ. An undrained shear strength gradient  $k = 1.70$  kPa/m is derived  
 261 from the critical state properties, giving a dimensionless strength heterogeneity coefficient  
 262  $kB/s_{um} \sim 16$ . Bearing capacity factors for rectangular foundations are available for soil  
 263 strength heterogeneity in the range  $0 \leq kB/s_{um} \leq 10$  [26], while solutions exist for the  
 264 appropriate  $kB/s_{um}$  but assuming plane strain geometry [27]. The outcome from extrapolating  
 265 a bearing capacity factor for the rectangular foundation geometry to the appropriate  $kB/s_{um}$  for  
 266 this example is similar to the plane strain solution (approximately 10% lower). For this  
 267 simulation the operative bearing pressure is taken as

$$268 \quad q_{op} = 0.3N_{cV}s_{um} = 0.3(15.85)(0.52 \text{ kPa}) = 0.3(8.25 \text{ kPa}) = 2.48 \text{ kPa} \quad (12)$$

270  
 271 By introducing the above value of  $q_{op}$  in Eq. (7), the increase of vertical stress  $\Delta\sigma'_v$  for the  
 272 characteristic points of each layer  $i$  can be determined. Assuming  $K_0$  compression conditions,  
 273 the Modified Cam Clay model has been employed to determine the vertical strain  $\varepsilon_v^i$  and the

274 void ratio  $e^i$  at each characteristic point. The overall vertical settlement of the soil can be  
 275 determined by assuming a constant vertical strain throughout each layer and by summing the  
 276 contribution of each layer to give:

277

$$278 \quad \delta_v = \sum_{i=1}^n \varepsilon_v^i h^i \quad (13)$$

279

### 280 **4.3 Foundation sliding stage**

281 Under operational conditions, the foundation is subjected to forward and backward cyclic  
 282 sliding. The concept of a sliding mudmat is such that the foundation is free to exceed by far  
 283 the displacement needed to mobilize the limiting soil-foundation shear resistance to sliding,  
 284  $\tau_{max}$ . The limiting soil-foundation shear resistance, which represents the (average) shear stress  
 285 cyclic amplitude which the sliding foundation is subjected to, is found as a proportion of the  
 286 foundation operative vertical stress  $q_{op}$ :

287

$$288 \quad \tau_{max} = q_{op} \tan \phi' = 1.08 kPa \quad (14)$$

289

290 where the friction between the mudmat and the soil is a function of the friction angle  $\tan \phi'$ ,  
 291 as was also detected in the experimental tests [2]. The shear stress distribution with depth  
 292 during full sliding can be predicted using the idealized elastic solution for a foundation on a  
 293 semi-infinite soil mass as a function of the shear stress on the surface  $\tau_{max}$  [6]:

294

$$295 \quad \tau = 2 \frac{\tau_{max}}{\pi} \left[ \text{atan} \frac{B}{2z} - \frac{z}{D} \text{atan} \frac{B}{2D} + \frac{B}{2z} \left( \frac{B^2}{4A^2} - \frac{B^2 + L^2}{4C^2} \right) \right] \quad (15)$$

296

297 The distribution of shear stress  $\tau$  normalized by the interfacial shear stress  $\tau_{max}$  with depth  
 298 during sliding is shown in Fig. 9b. The horizontal cyclic loading acting on the foundation has



299 been reproduced considering equivalent simple shear conditions; the shear stress cyclic  
300 amplitude  $\Delta\tau^{cyc}$  has been derived from Eq. (15) by assuming  $\Delta\tau^{cyc} = \pm \tau$ , where the sign  $\pm$  is  
301 implemented to include the direction of the loading cycles (forward or backwards). The  
302 simulations of the foundation sliding stage have been performed separately for the three  
303 constitutive models (MCC, STS and MSH) in order to investigate their capabilities. A total of  
304 40 back-and-forth cycles have been imposed in order to simulate the long-term behaviour of  
305 sliding foundations. The overall foundation settlement for this stage is determined using Eq.  
306 (13) above.

307

## 308 **5 Analysis results**

309

### 310 **5.1 Model performances**

311 Comparison of the predicted foundation vertical settlements normalised by the width of the  
312 foundation ( $\delta_v/B$ ) during the 40 imposed shearing cycles is presented in Fig. 10a for the MCC  
313 model, Fig. 10b the STS model and Fig. 10c for the MSH model. A direct comparison of  
314 evolution of the vertical settlements with number of loading cycles is also presented in Fig.  
315 10d. The predicted trends and magnitude of settlements differ considerably among these  
316 models. The MCC model predicts a normalized vertical displacement  $\delta_v/B = 9 \cdot 10^{-4}$  at the  
317 stable state, which is reached at the end of the very first shearing cycle with no further  
318 densification occurring with subsequent cycles since the load path lies within the now  
319 expanded yield surface (Fig. 10a). On the contrary the STS model predicts a much larger  
320 normalised vertical displacement,  $\delta_v/B = 1.5 \cdot 10^{-1}$  after 40 cycles (Fig. 10b). In this case, a  
321 stable state is not reached as the model does not accurately capture the rate of plastic strain  
322 hardening. The prediction of the MSH model lies between the MCC and STS model  
323 predictions outlined above, suggesting a normalized vertical displacement  $\delta_v/B = 4.4 \cdot 10^{-2}$

324 after 40 cycles which is gradually reached during the cyclic loading sequence (Fig. 10c).  
325 Settlement measured in the centrifuge test [2] is also reported in Fig. 10d for comparison with  
326 the three computations performed. It is clear that while the MSH model predicts qualitatively  
327 and quantitatively the closest behaviour to the experimental observation, the final vertical  
328 displacement of the foundation is slightly underestimated. This may be the result of  
329 limitations in the calibration exercise performed for experimental results on soil samples  
330 tested under different loading conditions (triaxial compression instead of simple shear) and  
331 under much larger confining pressure (up to two orders of magnitude).

332 Fig. 11 shows a comparison among the predictions of the three models for the settlement  
333 during the first slide. The normalized settlement predicted by the MCC model is the lowest  
334 among the models, which follows directly from the nature of the model (an isotropic  
335 hardening model). Moreover, the development of the vertical settlements is interrupted by the  
336 initial expansion of the yield surface, which occurs at  $N_{cyc} = 0.5$  (representing the first half of  
337 the foundation slide). The vertical settlements predicted by the STS and MSH models at  $N_{cyc}$   
338  $= 0.5$  are similar and the small difference is due to the slightly different hardening rule  
339 implemented in the models. During the first half loading cycle the soil stiffness in the two  
340 models is governed by the constitutive parameter  $\beta$ . At  $N_{cyc} = 1$ , the difference in the  
341 simulated vertical settlement increases as the dependence on the memory surface in the MSH  
342 model is activated leading to an increase in the plastic soil stiffness which inevitably reduces  
343 the magnitude of predicted vertical settlements. The plastic soil stiffness in the MSH model is  
344 now governed by the constitutive parameters  $\beta$  and  $\mu$ .

345 The difference between the three computations can be understood by analysing the predicted  
346 behaviours (Fig. 12), stress paths and evolutions of the yield surfaces (Fig. 13), and void ratio  
347 changes (Fig. 14) for the shallowest soil element considered. This soil element is indeed that  
348 subjected to the largest shear stress cycles and in turn shows the largest vertical strains.

349 The observed immediate achievement of the stable state for the MCC model is the result of  
350 the isotropic hardening expansion of the yield surface upon the first shearing cycle and the  
351 resulting fully elastic behaviour during the following shearing cycles imposed within the  
352 purely elastic region bounded by the yield surface (Fig. 12a and Fig. 13a). As expected no  
353 changes in void ratio are recorded after the first cycle, as shown in Fig. 14a.

354 The very large settlements predicted using the STS model (Fig. 12b) are the result of the  
355 imposed kinematic hardening to the yield surface and the lack of any mechanism to record  
356 that the soil has been subjected to the same shearing conditions in previous cycles. In fact,  
357 during cyclic shearing, the yield surface moves up and down kinematically as shown in Fig.  
358 13b and the only difference between subsequent cycles is a slight expansion of the bounding  
359 surface caused by the progressive soil densification. The plastic soil stiffness, which is  
360 affected by the distance between the current stress state and the image stress on the bounding  
361 surface (the quantity  $b$  in Fig. 5b), only slightly increases during cyclic shearing and a stable  
362 state cannot be reached. Actually, the soil continues to compress indefinitely to reach  
363 unreasonable values such as a vertical strain  $\varepsilon_v = 1.00$ . The soil would continue to compress  
364 unrealistically if further cyclic shearing were to be imposed.

365 The response of the MSH model lies in between the MCC and STS model responses as  
366 shown by the stress-strain behaviour in Fig. 12c. For this model, the yield surface still moves  
367 up and down kinematically as shown for the STS model in Fig. 13b, but the introduction of  
368 an evolving memory surface (shown in Fig. 13c) allows the progressive stiffening during  
369 repeated loading to be captured. One can interpret the memory surface as a record of the  
370 current fabric of the soil, describing the range of stresses that can be imposed without major  
371 disruptions to particle arrangement. Thus, the memory surface bounds a region of increased  
372 stiffness and its progressive increase during cyclic shearing allows a progressive increase of  
373 the plastic soil stiffness. In this case a stable state is gradually reached with increased number

374 of cycles. Similarly to the STS model, the MSH model predicts a compression of the soil  
375 element to a void ratio well below the critical state line (Fig. 14c) but the progressive soil  
376 stiffening prevents the soil from reaching inadmissible states. The independency of the soil  
377 volumetric response from the corresponding critical state value during cyclic loading has  
378 been confirmed experimentally by several studies on granular soils (e.g. [28-29]). It should  
379 also be noted that the response of the STS and MSH model during the first shearing cycle are  
380 similar and the additional memory surface of the MSH model affects only the soil behaviour  
381 under unloading-reloading conditions.

382 Among the three analyses, the computation using the MSH model is the only one  
383 qualitatively similar to the experimental results obtained by Cocjin et al. [2] with a stable  
384 state reached progressively during cyclic loading. Using material parameters and boundary  
385 conditions available in the literature [2, 22], a reasonable quantitative fit to observed  
386 experimental results [2] is also achieved with the MSH model.

387

## 388 **5.2 Sensitivity analysis of MSH model hardening parameter, $\mu$**

389 In this section a numerical exercise is provided to assess the sensitivity of the MSH model  
390 parameter,  $\mu$ . This parameter affects the magnitude of the soil hardening modulus, thus of soil  
391 stiffness, when the stress state lies within the memory surface. The larger the parameter, the  
392 stiffer the soil response under cyclic loading but the slower the expansion of the memory  
393 surface during progressive cycles. If this parameter is set to zero the STS model is recovered.  
394 Fig. 15 indicates the effect of varying the constitutive parameter  $\mu$ , introduced in Eq. (1), and  
395 shows that the predicted vertical settlement at the stable state increases by decreasing the  
396 value of the parameter  $\mu$ . The results from the three simulations are compared directly in Fig.  
397 15d where the accumulated vertical displacements are plotted against the number of cycles;  
398 the final displacement measured experimentally by Cocjin et al. [2] is also shown and it

399 seems that a value of the parameter  $\mu = 8$  provides a good fit to the final displacement at the  
400 stable state. It should be noted that the value that fits the experimental data ( $\mu = 8$ ) is quite  
401 close to that obtained from the model calibration exercise ( $\mu = 11.5$ ).

402 The foundation set-down settlement is well reproduced by the numerical analysis;  
403 experimentally the measured settlement is 0.081 m while the calculated settlement from the  
404 simulation is 0.077 m; the foundation set-down displacement is calculated using the MCC  
405 model and assuming  $K_0$ -consolidation conditions.

406 Using the best-fit simulation to the experimental data it is possible to analyse the predicted  
407 trend of void ratio and settlement with depth. Fig. 16a presents the profile of void ratio with  
408 normalized depth predicted at the end of the three stages (geostatic, foundation set-down and  
409 cyclic shearing), while Fig. 16b presents the trends of displacement  $\delta_v$ , normalised by the  
410 width of the foundation  $B$  for each loading stage. From Fig. 16b, it is clear that while the  
411 foundation set-down induces less settlement in the soil than the cyclic shearing stage, it  
412 affects layers to a greater depth. The foundation set-down stage affects deeper soil layers and  
413 this is confirmed by the percentage of vertical displacements (72%) occurring within a depth  
414 of  $0.5B$ . By contrast, the cyclic shearing stage appears to be a very superficial mechanism  
415 which just affects the soil to a maximum depth of  $0.5B$ ; in fact almost 100% of vertical  
416 settlements take place in the top 3 m of soil ( $0.6B$ ). These findings are related to the stress  
417 distribution from the assumed elastic solution for the foundation set-down stage (Fig. 9a) and  
418 the sliding stage (Fig. 9b). The zone of influence of horizontal cyclic loading is predicted to  
419 be limited to a depth of approximately half the foundation breadth, in good agreement with  
420 the experimental results [2].

## 421 **6 Parametric study**

422 In this section a parametric study is presented to assess the effect on whole-life settlement of  
423 sliding foundations after 40 lateral loading cycles as a function of soil strength heterogeneity

424 and foundation aspect ratio. The simulations were undertaken using the Memory Surface  
425 Hardening model calibrated using the input values given in Table 2, but with the best-fit  
426 hardening value presented above ( $\mu = 8$ ). The strength properties, overburden stresses and  
427 foundation weights considered in the parametric study are summarised in Table 3. The soil  
428 strength properties were obtained using Eqs. (9) to (11), imposing the Modified Cam-Clay  
429 constitutive parameters presented in Table 2 and changing the overburden stress. Figure 17  
430 (a) shows the long term normalised foundation settlement with the number of cycles of  
431 sliding on soils with different undrained shear strength profiles. Shear strength profiles of  
432  $kB/s_{um} = 0, 20$  and  $100$  were assumed in this assessment. The vertical displacement after 40  
433 cycles seem rather comparable for a shear strength profile of  $kB/s_{um} = 20$  and  $100$ , where after  
434 40 cycles a normalised displacement of  $0.058$  and  $0.055$  is calculated. The normalised  
435 displacement for a soil strength  $kB/s_{um} = 0$  is rather limited as a consequence of the large  
436 overburden stress, which implies significant soil compression during the foundation set down  
437 stage, as shown in Table 3. Figure 17 (b) shows the accumulated normalised vertical  
438 displacement of sliding foundations with different aspect ratio  $B/L$ . The simulations show that  
439 the accumulated normalised displacement increases as the aspect ratio  $B/L$  decreases.

440 Table 3: Soil parameters for parametric study of Figure 17 (a)

$kB/s_{um}$	$s_{um}$ (kPa)	$k$ (kPa/m)	Overburden stress, $\Delta\sigma'_v$ (kPa)	$q_{op}$ (kPa)
0	112	2.2	400	195
20	0.42	1.7	1.50	2.18
100	0.08	1.67	0.30	1.27

441

442 **7 Conclusions**

443 In this paper a numerical procedure has been proposed to estimate the whole-life settlements  
444 of tolerably mobile subsea foundations under cycles of horizontal shearing. The approach is  
445 based on the validated assumption that a drained soil response is an appropriate proxy for  
446 alternating stages of undrained sliding and consolidation.

447 A one-dimensional model was developed to represent stress and volume changes beneath a  
448 mat foundation.

449 Three different constitutive models were used to simulate the soil response to the whole life  
450 operation of a sliding foundation: the Modified Cam Clay model, the Severn-Trent Sand  
451 model and the Memory Surface Hardening model. Initially the models were calibrated  
452 against a drained cyclic triaxial test available in the literature. The Modified Cam Clay model  
453 was adopted to simulate the soil consolidation and the foundation set-down stages, assuming  
454  $K_0$ -consolidation stress path. The foundation sliding stage was then performed by simulating  
455 40-cycles of simple shear; the three constitutive models were adopted to calculate the vertical  
456 settlements of the foundation. The Severn-Trent Sand model struggled to reproduce the  
457 typical stable state that is observed in soils under cyclic loading conditions while the  
458 Modified Cam Clay model predicted just elastic strains under drained cyclic loading. The  
459 Memory Surface Hardening model was able to reproduce both plastic deformations under  
460 cyclic loading and an approach to the stable state.

461 The following aspects were observed:

462

463 - The vertical displacement related to the foundation set-down stage could be well  
464 reproduced by the Modified Cam Clay Model.

465 - The maximum density condition (at the steady state) could be well predicted by the  
466 Memory Surface Hardening model but not the Modified Cam Clay Model and Severn Trent  
467 Sand models.

468 - The numerical analysis confirmed that the foundation sliding mechanisms influences  
469 just the near surface soil.

470

471 The Memory Surface Hardening model was demonstrated to successfully calculate the  
472 vertical settlements of a sliding foundation on normally consolidated kaolin clay when  
473 compared with results from an available centrifuge test. The model applicability should be  
474 investigated further for different foundation and soil conditions before being applied outside  
475 the conditions considered in this study. Nonetheless, the study has indicated that this class of  
476 model has the potential to capture the essential components of whole-life settlements of  
477 tolerably mobile subsea foundations through the assumption that a drained soil response is an  
478 appropriate proxy for alternating stages of undrained sliding and consolidation.

479

480

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494

## 495 **9 Notation list**

496

### **Symbol Description**

$A$	Geometrical distance to calculate the stress distribution beneath the foundation
$A_d$	Flow rule multiplier
$B$	Foundation width
$C$	Geometrical distance to calculate the stress distribution beneath the foundation
$D$	Geometrical distance to calculate the stress distribution beneath the foundation
$L$	Foundation length
$b$	Distance between the current stress state $\sigma$ and the conjugate one on the bounding surface $\sigma^B$
$b^M$	Distance between the current stress state $\sigma$ and the conjugate one on the memory surface $\sigma^M$
$b_{max}$	Maximum value of $b$
$\beta$	Parameter controlling the amount of settlement in the first slide
$d$	Dilatancy flow rule
$\delta_v$	Vertical displacement
$e$	Void ratio

$e_{CSL}$	Intercept of the critical state line in $e$ - $\ln p'$ space at $p'=1$ kPa
$\varepsilon_v$	Vertical strain
$f_Y$	Yield surface
$f_B$	Bounding surface
$f_M$	Memory surface
$h$	Layer thickness
$H$	Hardening modulus
$k$	Undrained shear strength gradient
$k^*$	Parameter controlling the relationship between state parameter and available strength
$k_d$	Stress-dilatancy parameter
$k_0$	Lateral earth pressure
$M_{cv}$	Critical state stress ratio for compression
$N_{cV}$	Bearing capacity factor
$N_{cyc}$	Number of cycles
$p'$	Effective mean stress
$p'_c$	Consolidation pressure
$p_{ref}$	Reference pressure (=100 kPa)
$q$	Deviatoric stress
$q_{op}$	Foundation vertical pressure
$q_u$	Ultimate bearing capacity
$R$	Ratio of sizes of yield surface and bounding surface
$s_{um}$	Undrained shear strength at the surface

$z$	Depth
$\gamma'$	Effective unit weight
$\eta$	Stress ratio
$\kappa$	Slope of re-compression line in the e-ln p' space
$\lambda$	Slope of the critical state line in e-ln p' space
$\mu$	Constitutive parameter affecting the MSH model response in cyclic conditions
$\nu$	Poisson's ratio
$\sigma$	Stress state
$\sigma^B$	Conjugate stress point on the bounding surface
$\sigma^M$	Conjugate stress point on the memory surface
$\sigma'_v$	Effective vertical stress
$\varsigma$	Constitutive parameter affecting the contraction of the memory surface
$\tau_{max}$	Maximum shear stress under cyclic loading
$\psi$	State parameter
$\Delta\tau_{cyc}$	Cyclic amplitude in each soil layer

497

498

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572

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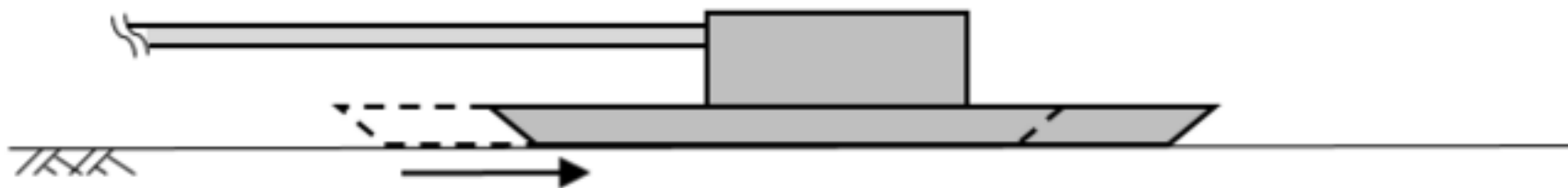


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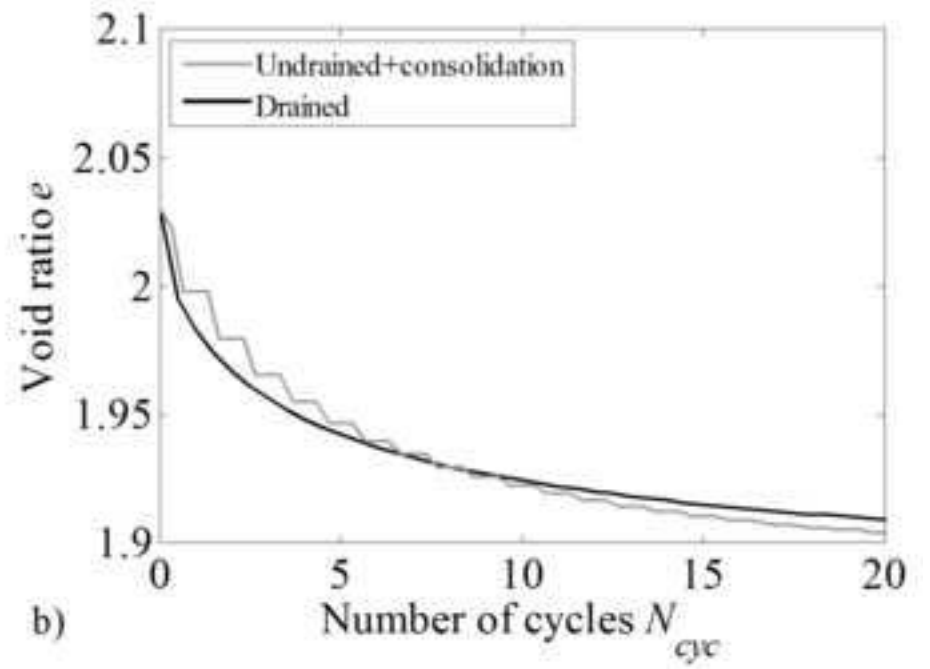
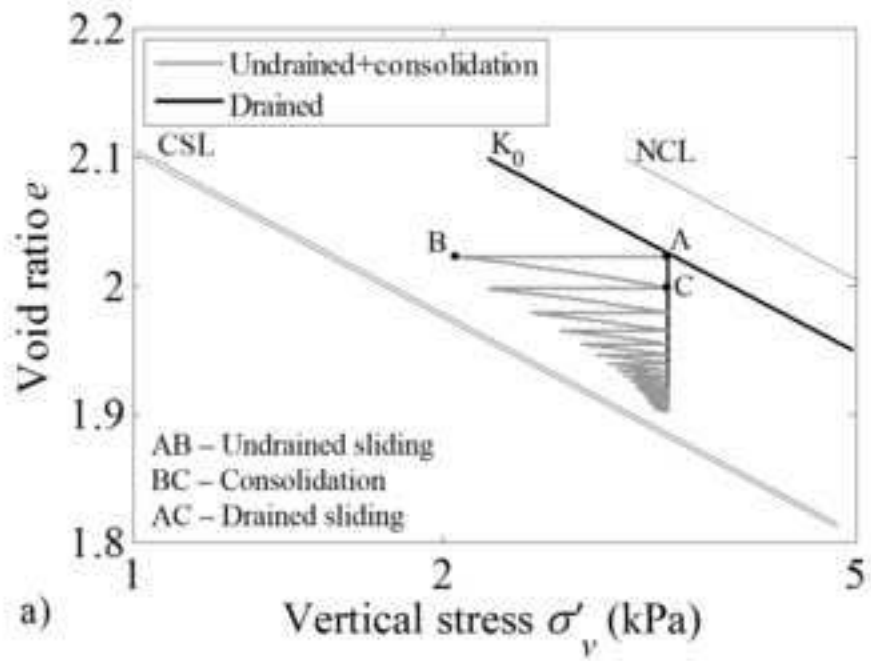




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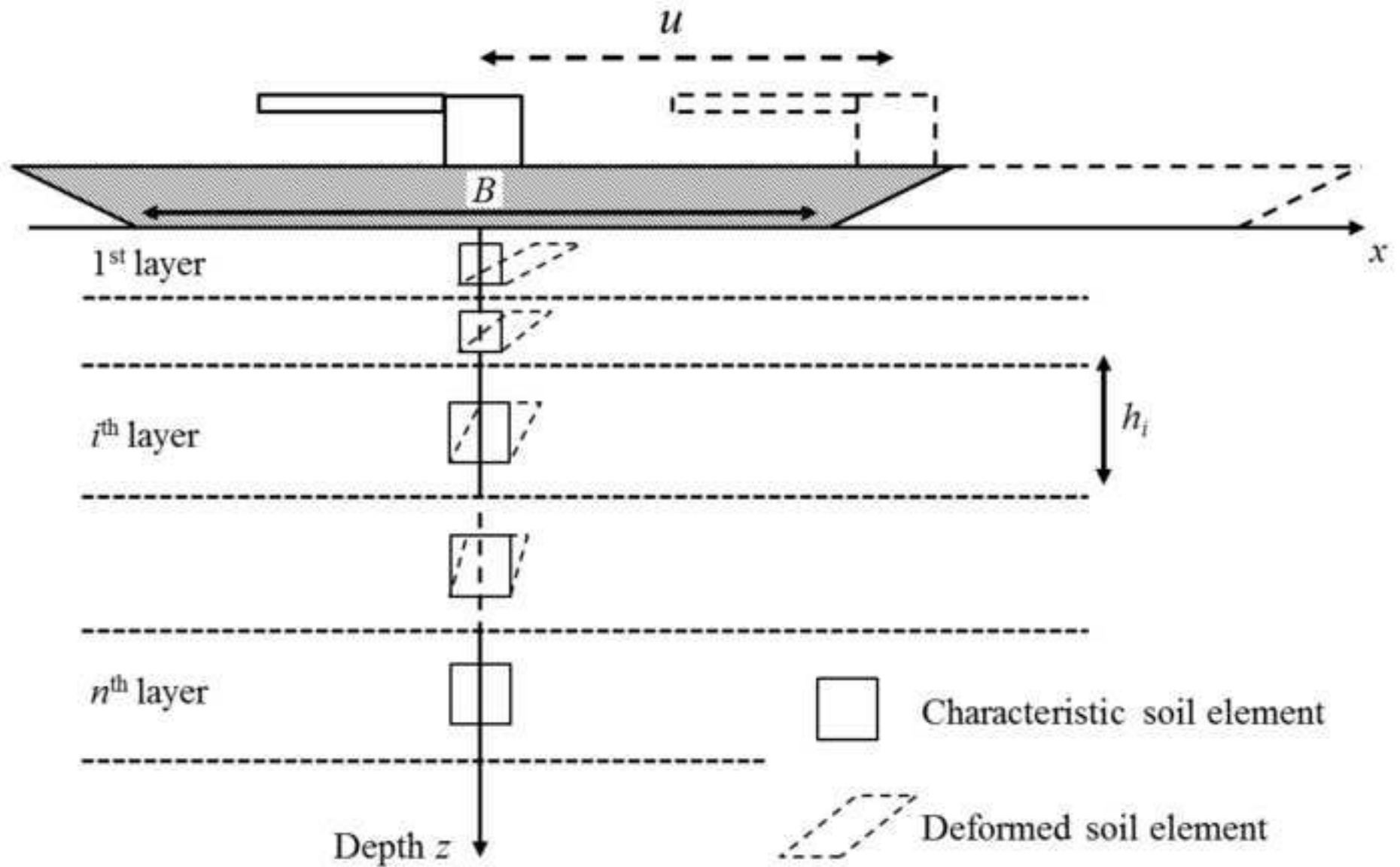


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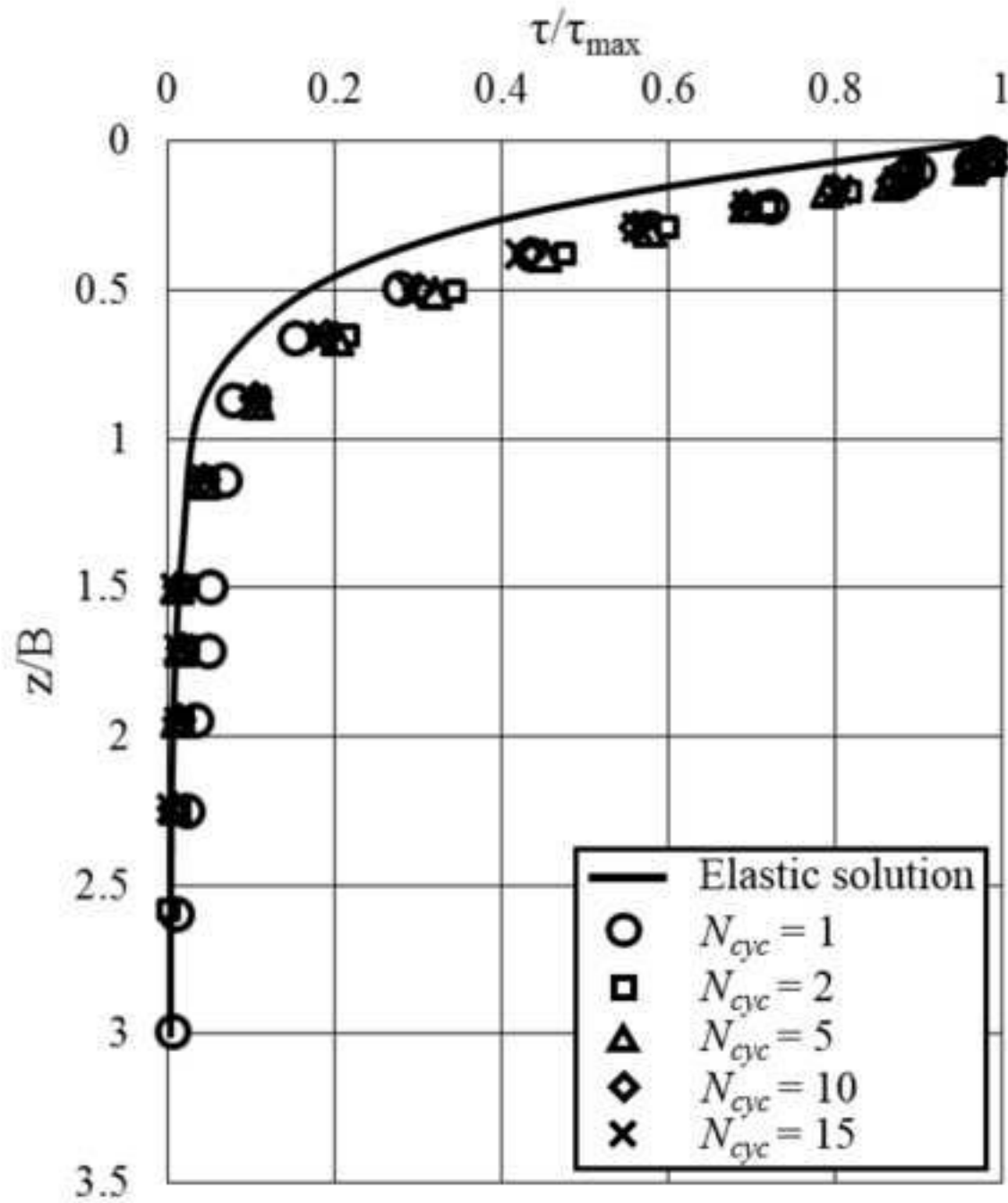


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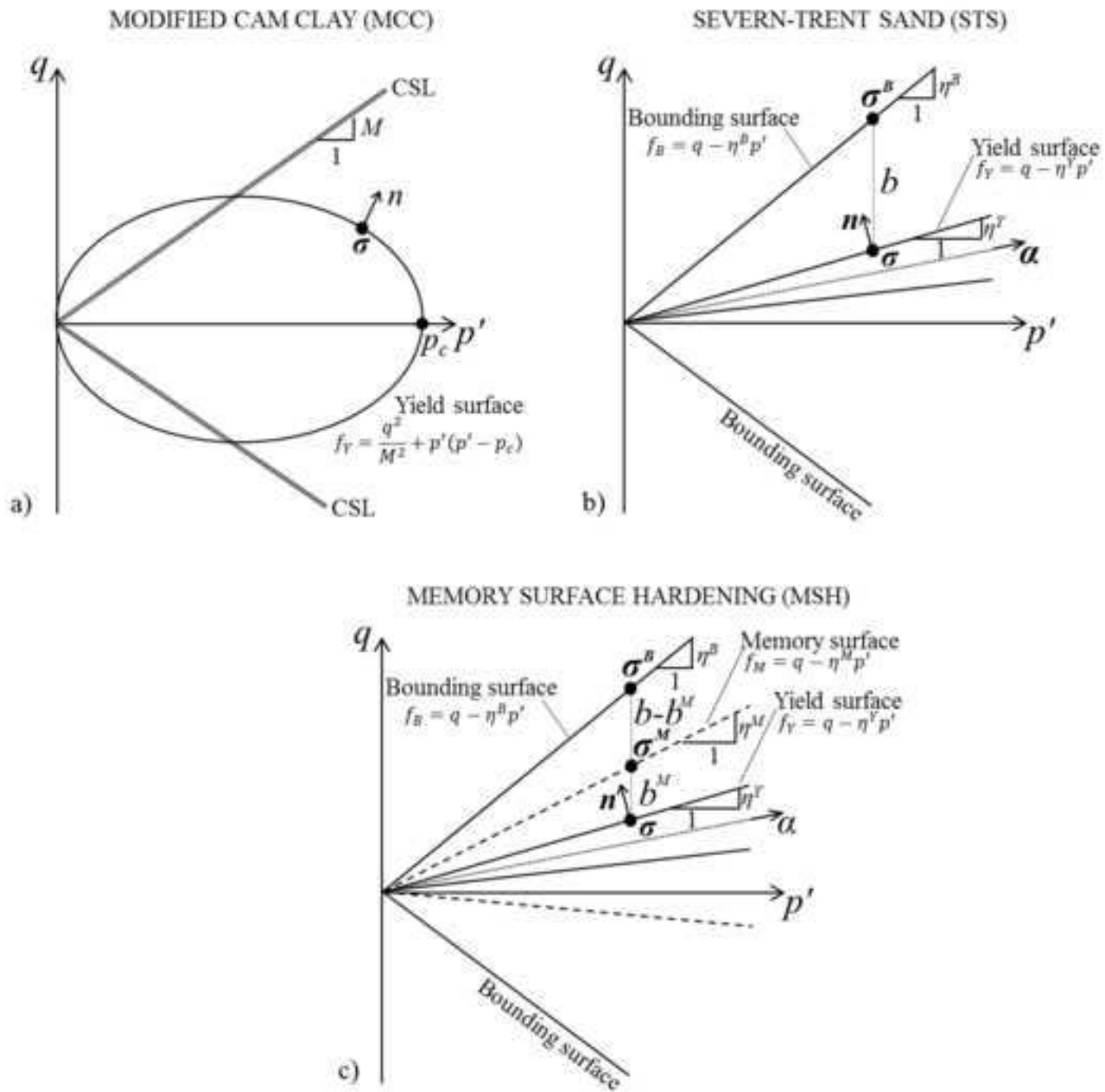


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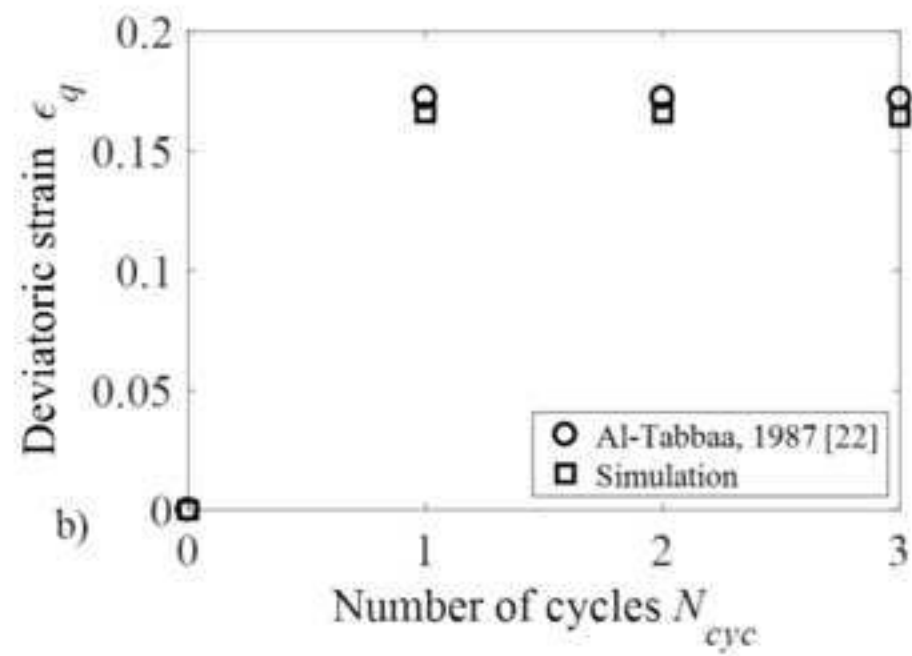
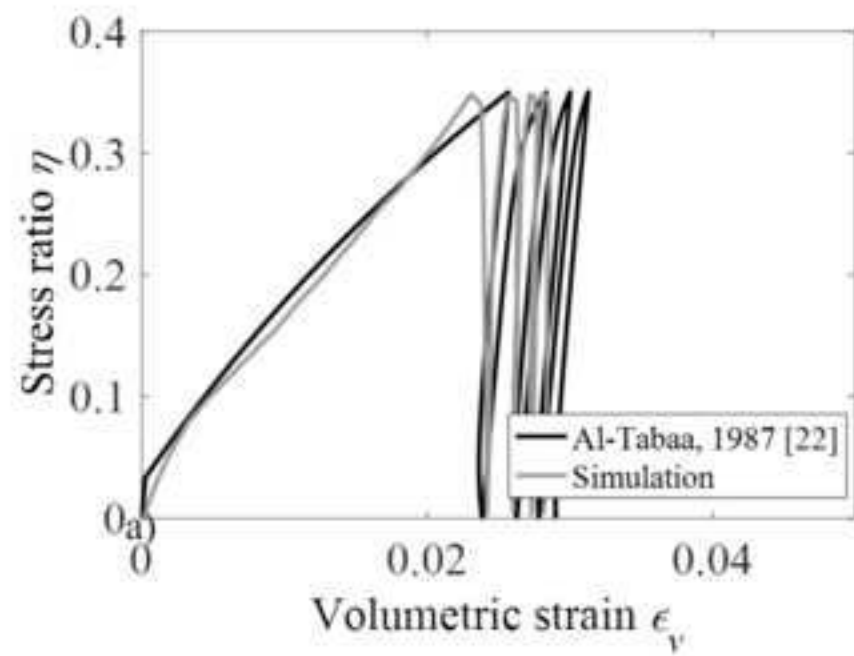


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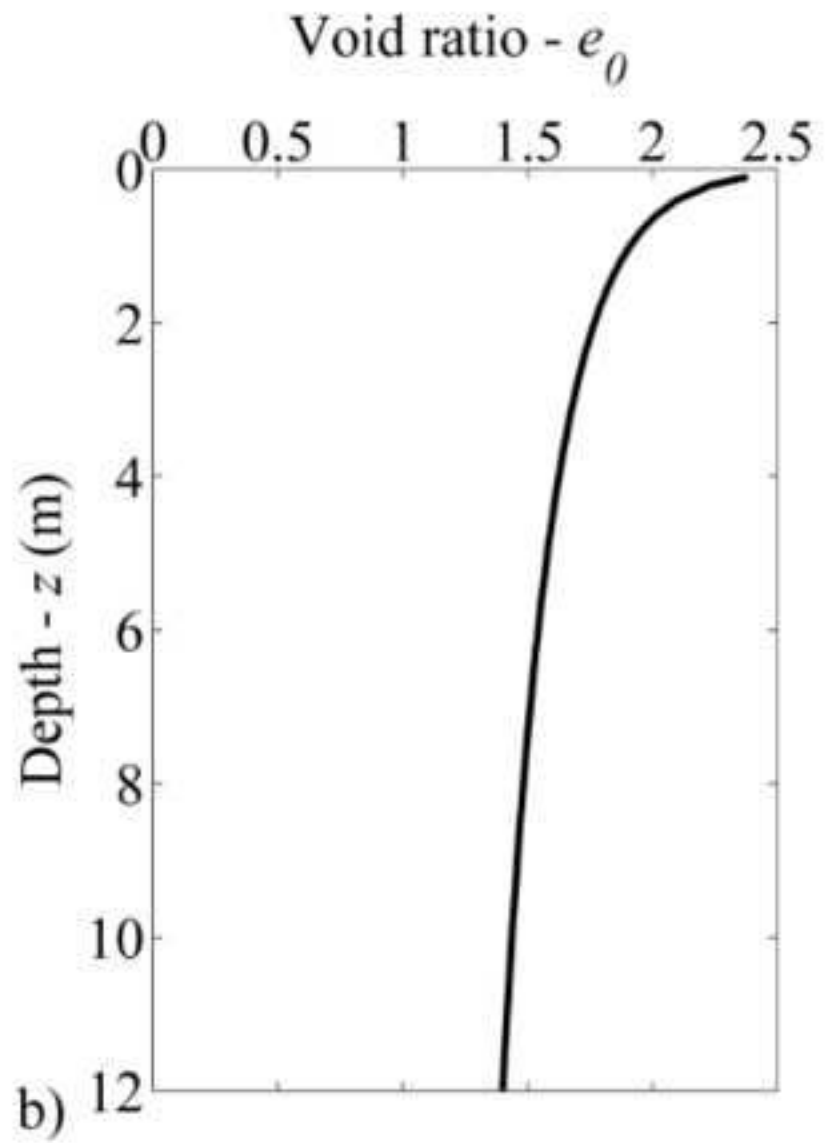
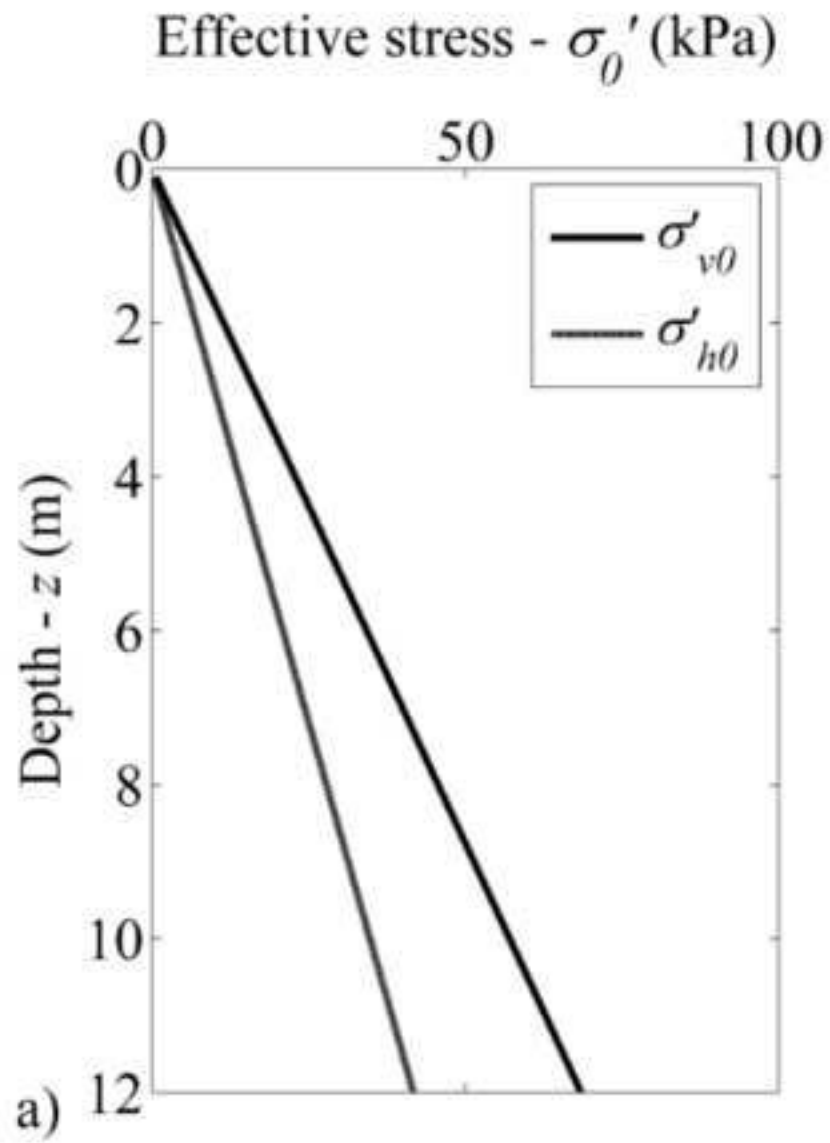


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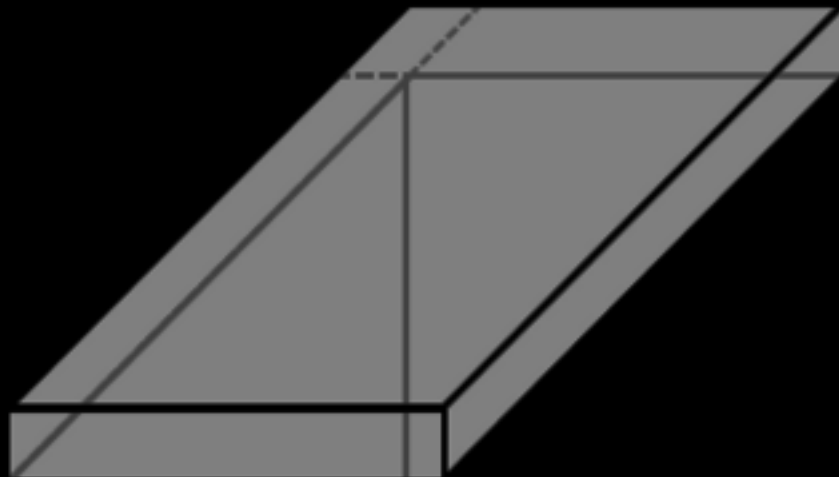


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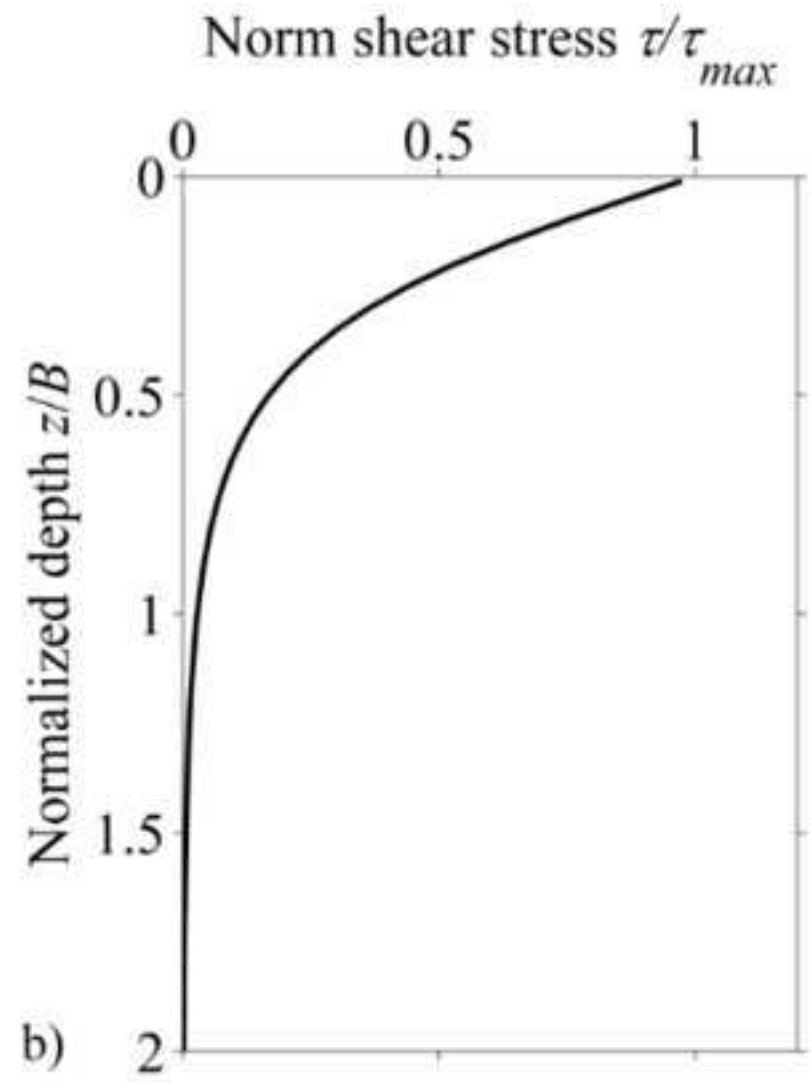
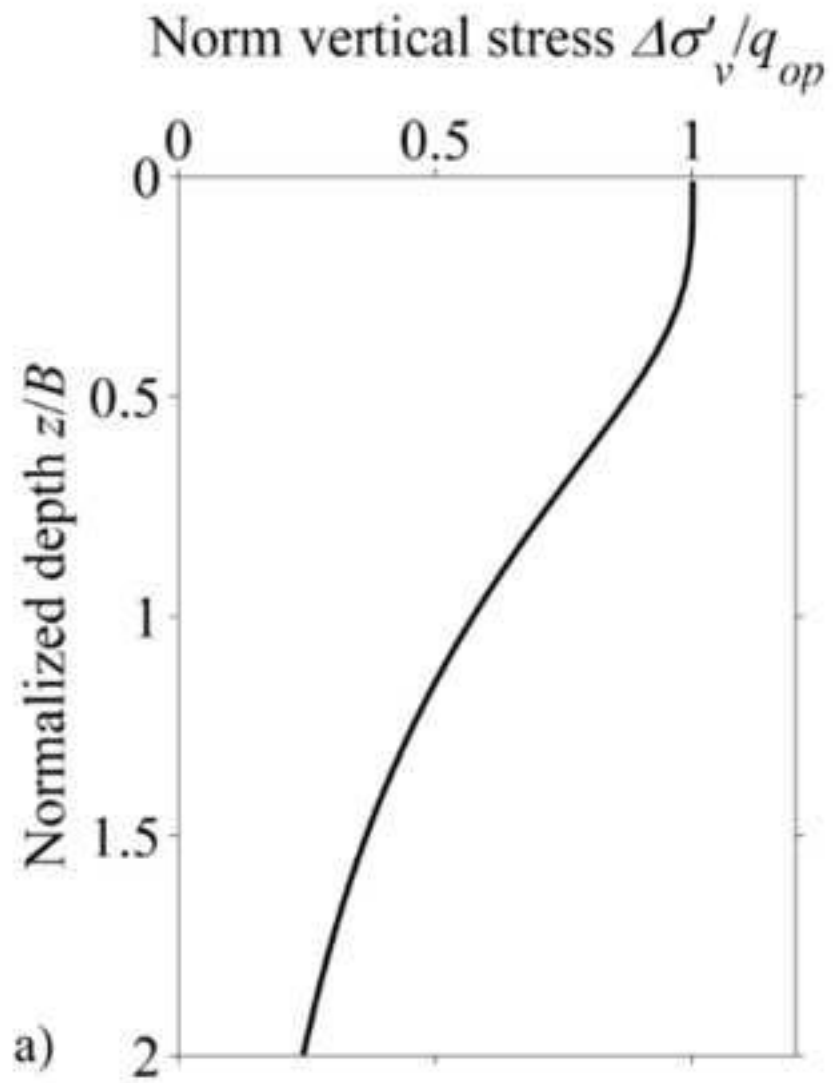


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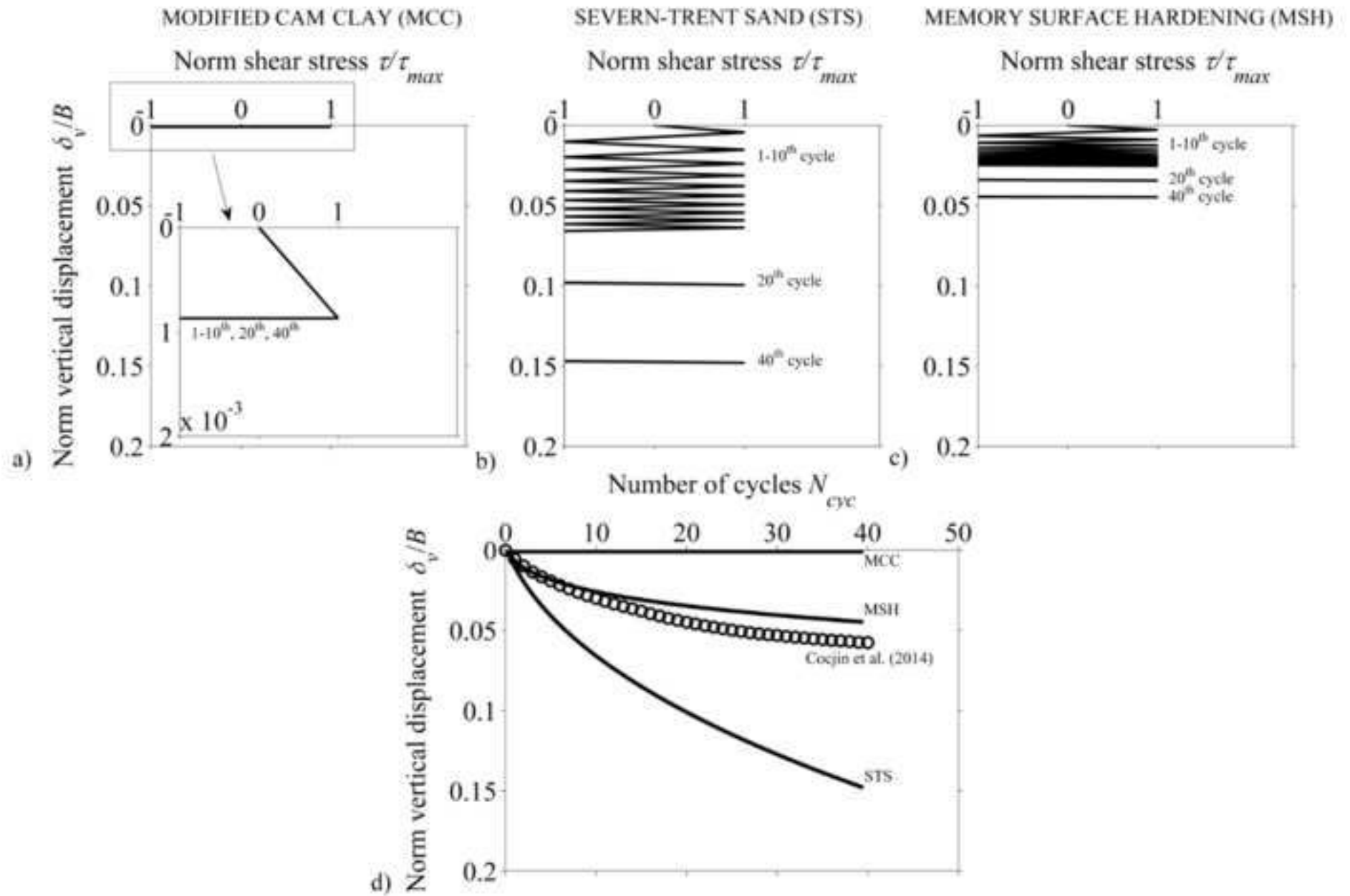




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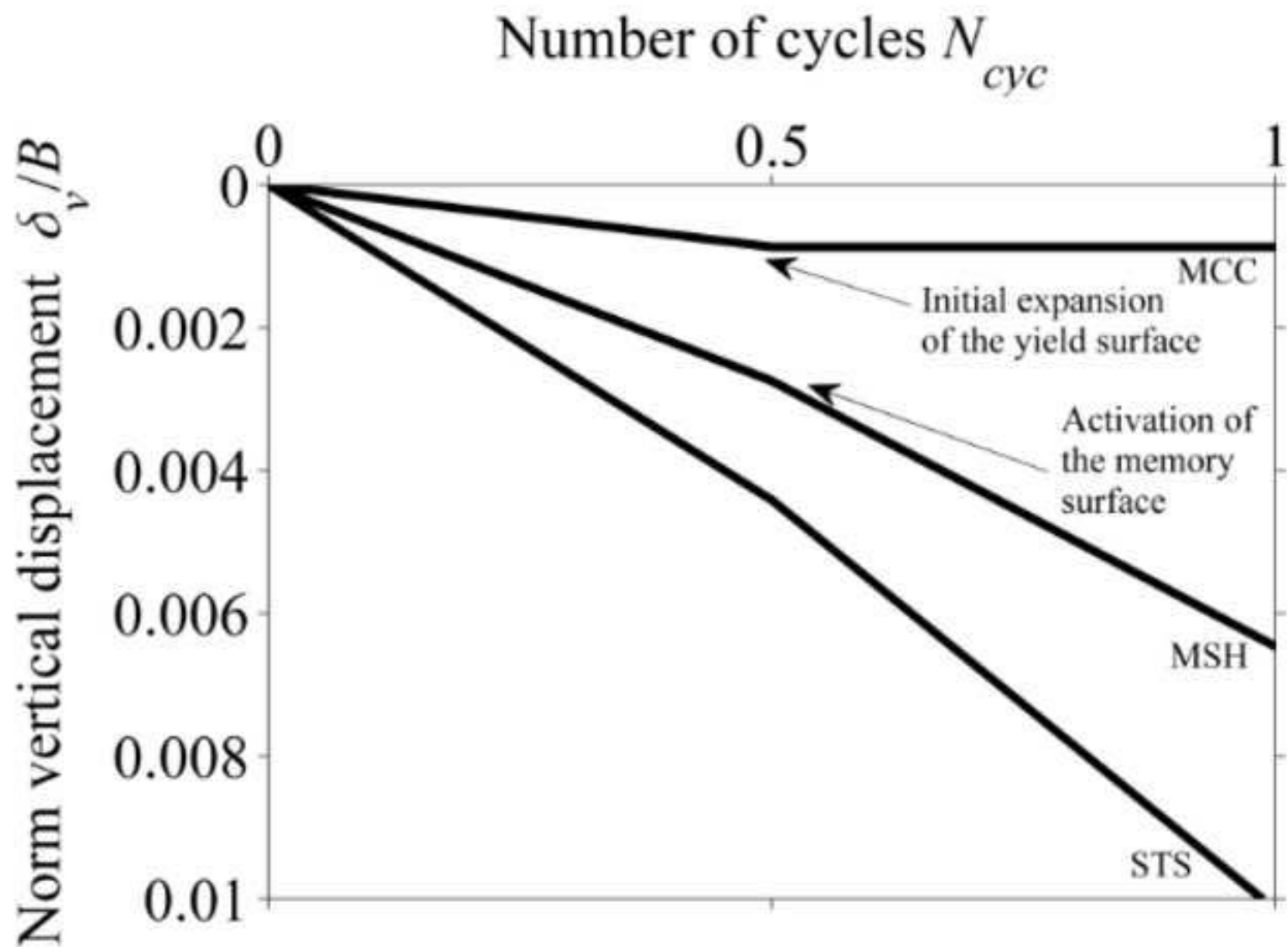


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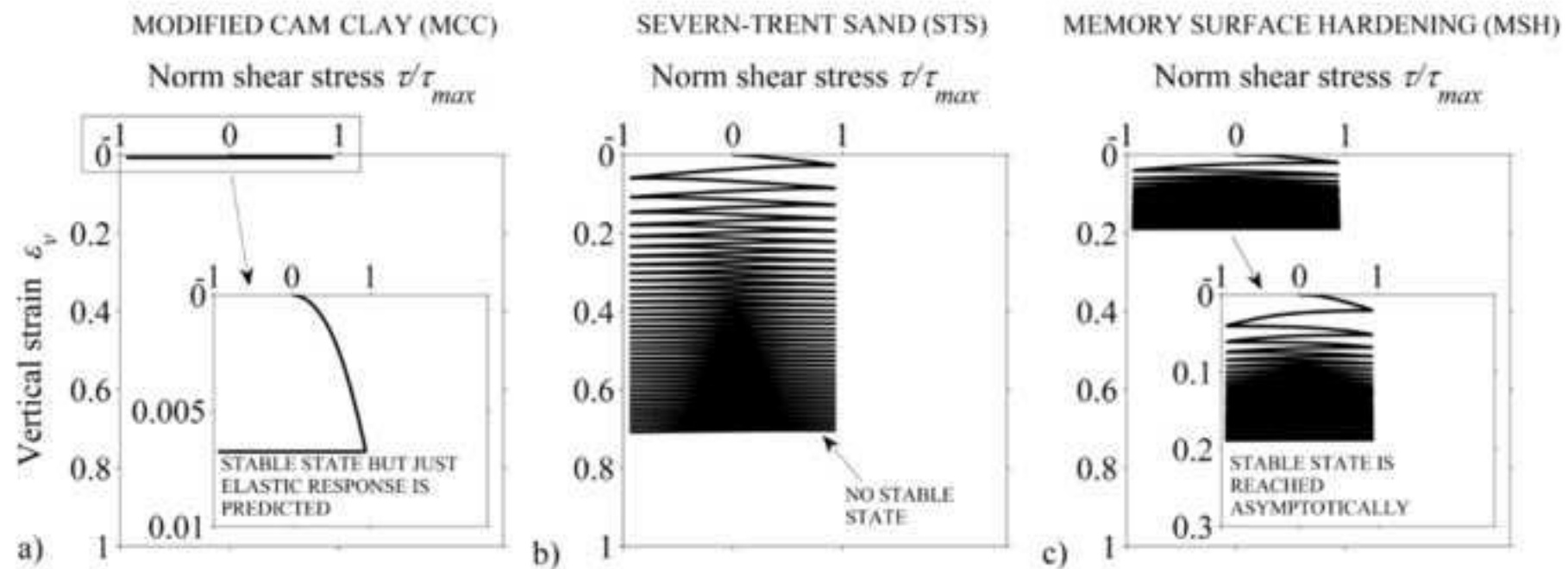


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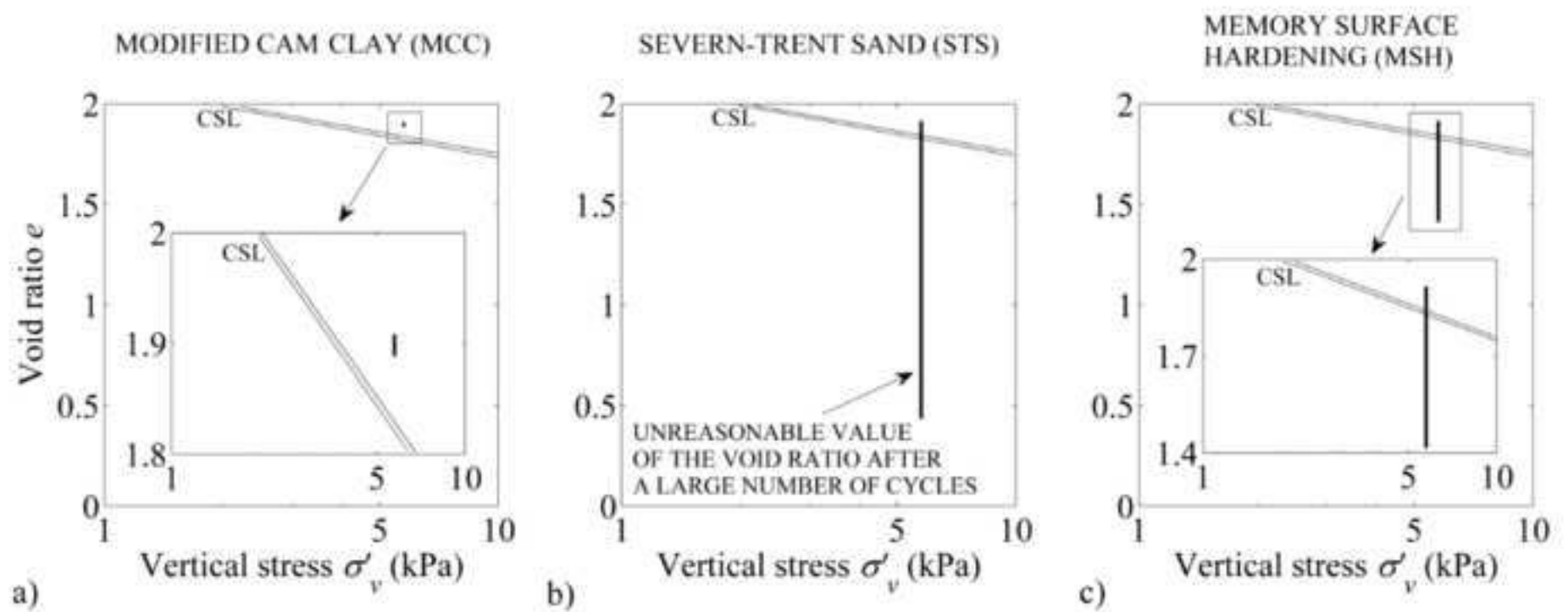


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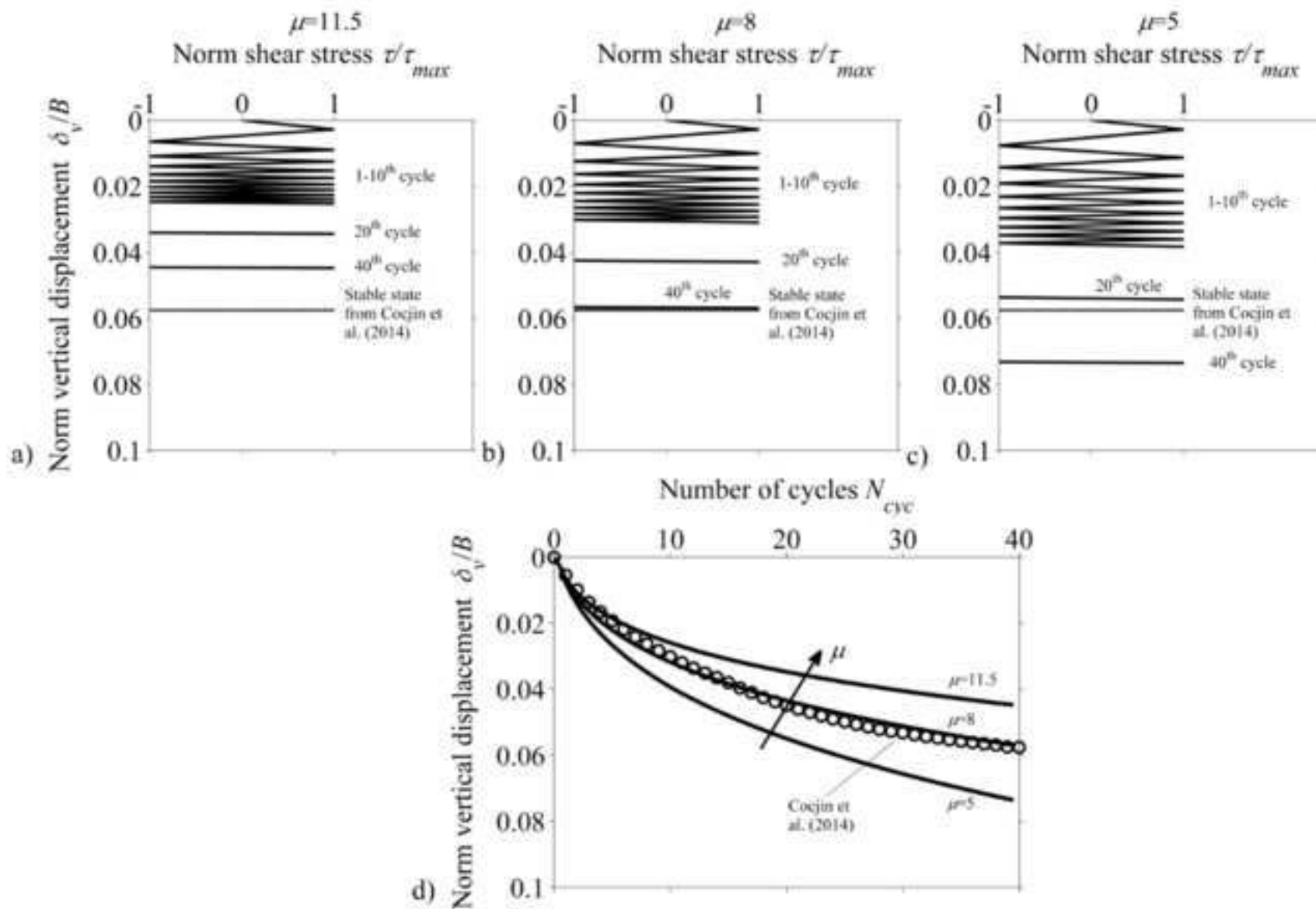


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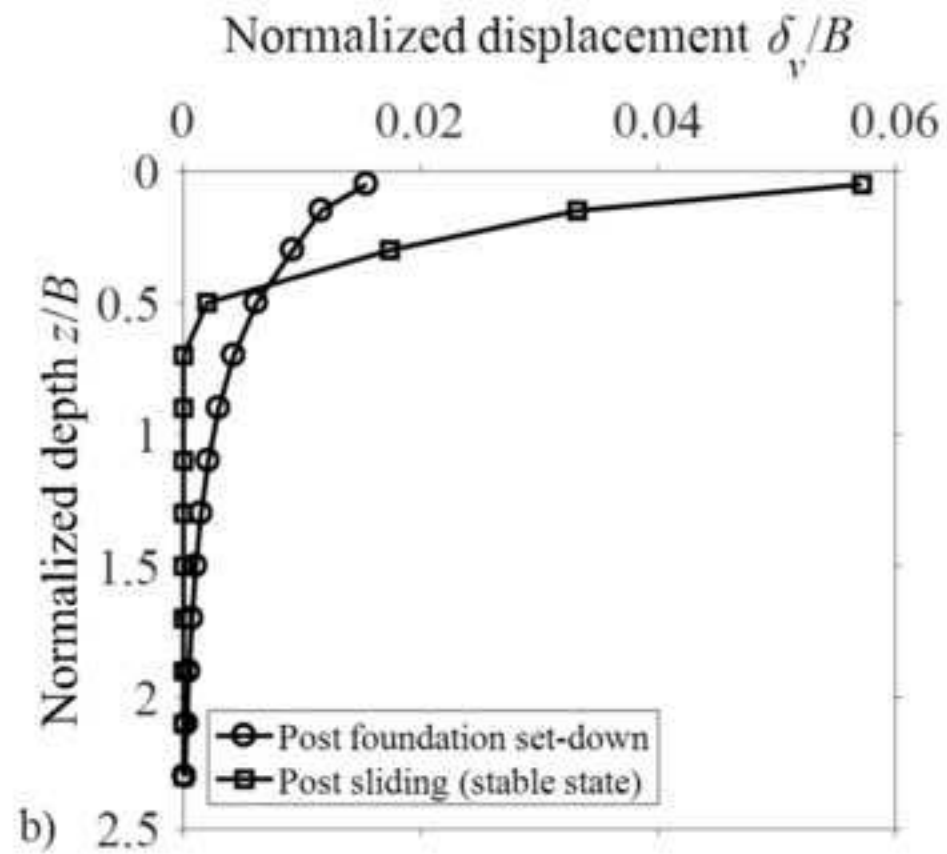
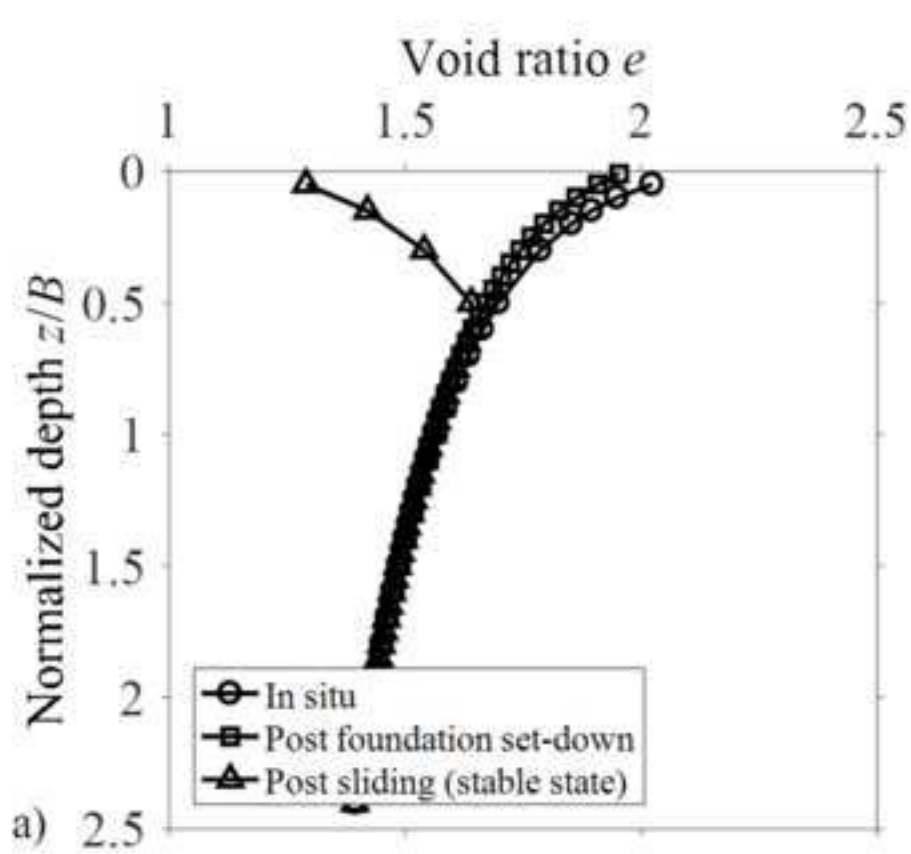
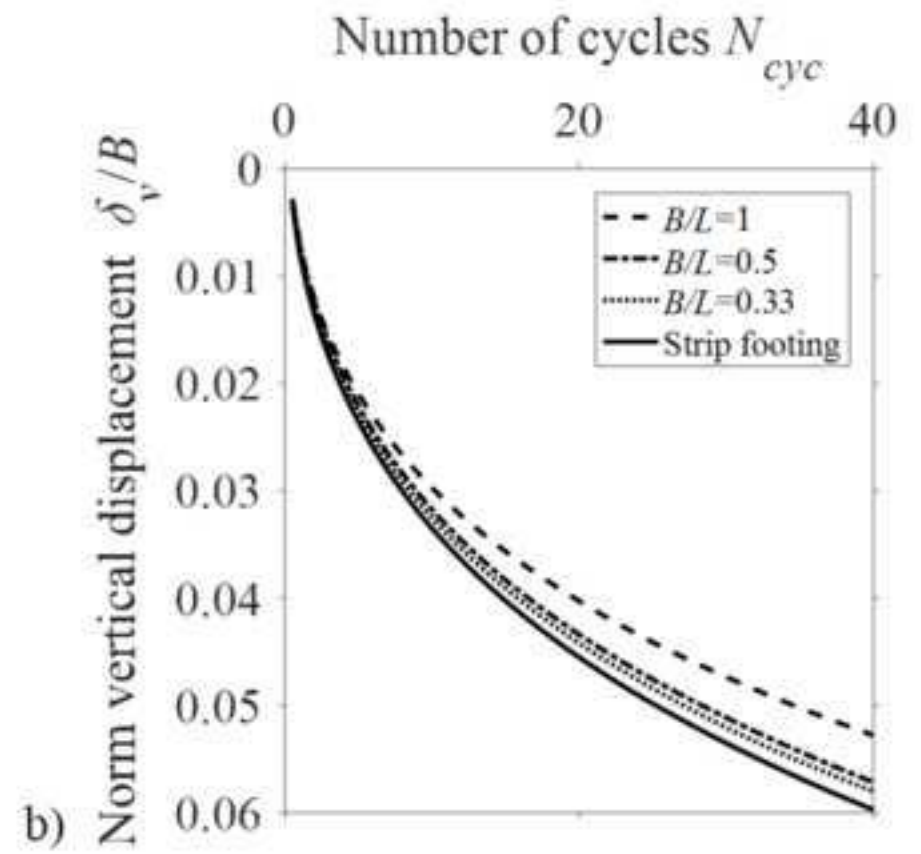
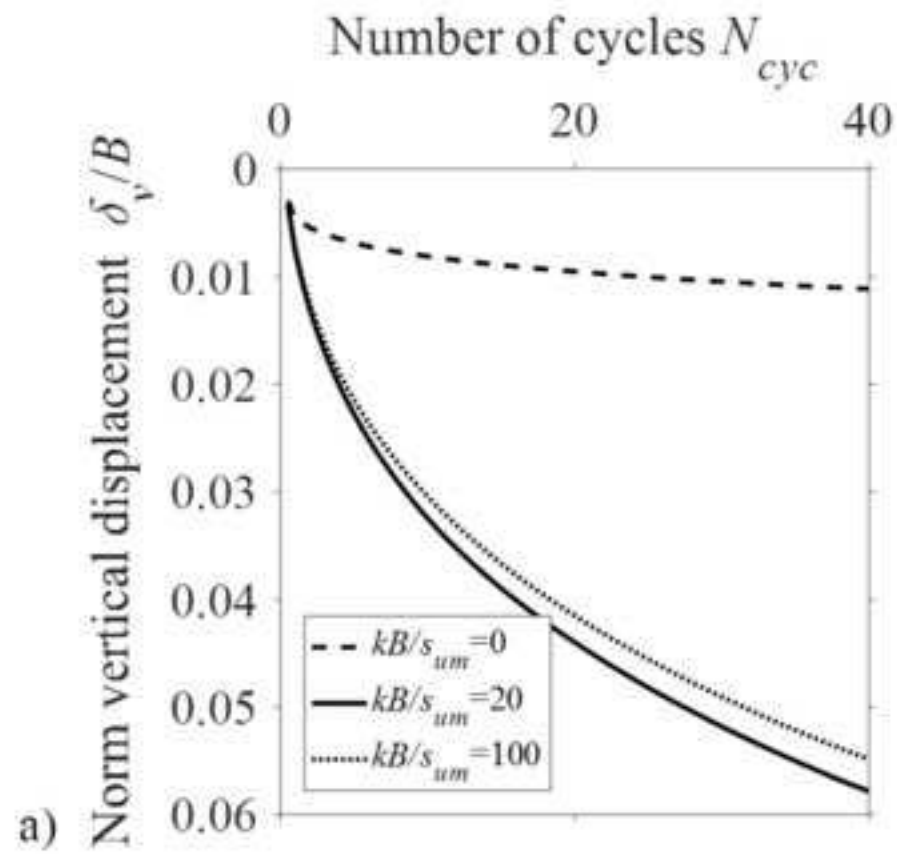


Figure 17

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Fig. 2: Comparison of (a) volumetric response and (b) variation of void ratio with the number of cycles of a soil element close to the interface between the sliding mudmat and the soil ( $z/B=0.05$ , with  $z$  being the depth and  $B$  being the width of the foundation), subjected to cyclic simple shear under constant total vertical stress by imposing either undrained loading with intervening consolidation or drained loading conditions, using the Modified Cam Clay constitutive soil model [5].

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